PROBABILISTIC DESIGN METHOD OF LEVEE AND FLOODWALL HEIGHTS FOR THE HURRICANE PROTECTION SYSTEM IN THE NEW ORLEANS AREA

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1. INTRODUCTION

The Hurricane Protection System (HPS) around the city of New Orleans will be significantly revised in the coming years after the flood disasters during Hurricane Katrina and Rita in 2005 (Figure 1). The entire system is re-designed and new closure structures are included to protect this city against hurricane threats in the future. Right now, the system is designed to protect the flood prone areas against a 1% hurricane event, better known as 100-year level of protection. This system-wide level of protection has to be effective in 2011 and is considered as a first step towards a better flood protection of the low-lying areas in the Mississippi river delta region. Ongoing planning efforts from US Army Corps of Engineers (USACE) and the State of Louisiana evaluate the need and possible measures for higher safety levels to protect populated areas.

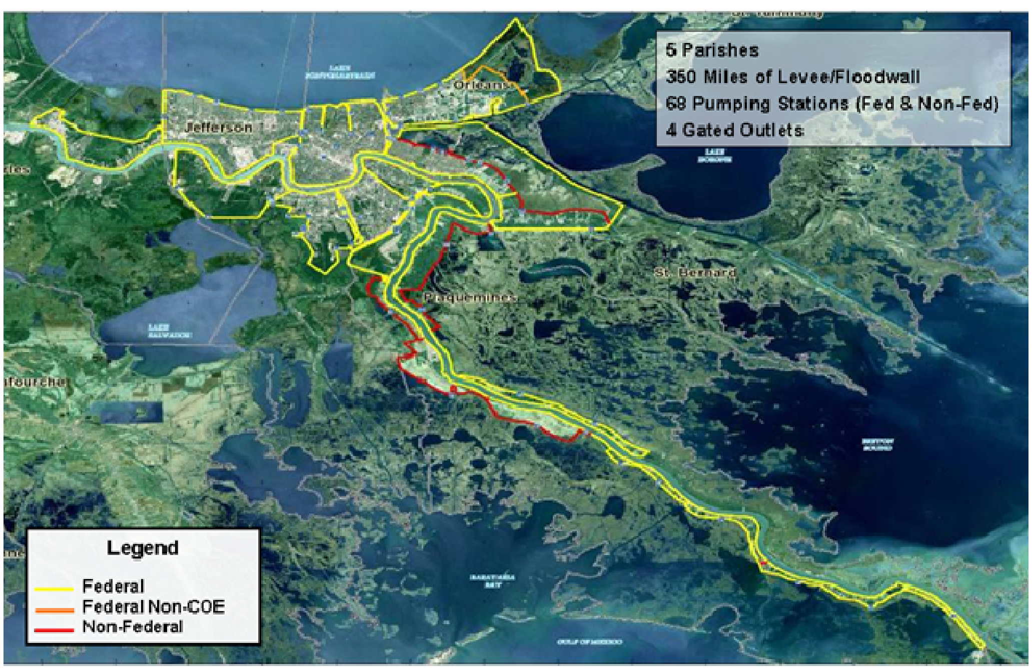

Figure 1. Hurricane Protection System (source: USACE)

The FEMA guidance for levee certification for coastal levees says that "the freeboard must be established at one foot above the height of the 1% wave or the maximum wave runup (whichever is greater) associated with the 100-year still water surge elevation at the site" (FEMA, 2007). The extra 1ft is added to account for uncertainties in the design variables. A disadvantage of this deterministic approach is that the uncertainty in the various hydraulic variables is not taken into account explicitly. The Corps guidance on certification is for a probabilistic approach to be followed similar to the approach used in Corps planning studies; yet, the procedure for coastal levees and floodwalls has not been well defined.

This paper presents a simple probabilistic approach that explicitly takes into account the uncertainties in the hydraulic design parameters. Starting point of this approach are the hydraulic variables near the flood defense structure. A limitation of the presented approach at this moment is that the correlation between the various hydraulic variables is not taken into account. Furthermore, the presented approach is limited to the hydraulic design of these structures for the time being. However, the approach could be extended for other uncertainties (geotechnical, structural) as well.

This paper is organized as follows. In Section 2 the current design guideline and the new design criterion for the HPS are discussed. Next, the analysis to include hydraulic uncertainty explicitly in the design method is explained in Section 3. Several test cases of the new design approach follow in Section 4. This paper closes with discussion and conclusions in Section 5.

2. NEW DESIGN CRITERION

The current FEMA certification guideline says that *"the freeboard must be established at one foot above the height of the 1% wave or the maximum wave runup (whichever is greater) associated with the 100-year still water surge elevation at the site"* (FEMA, 2007). Apart from discussion about the exact definition of the maximum runup or the 1% wave, this guideline was not considered to be sufficient for the hydraulic design of the HPS in the New Orleans area. Not the wave runup but the overtopping rate is probably the governing factor in the erosion of the backside slope. This overtopping rate (or even better the associated current velocity) will eventually determine the erosion of the backside of the levee/floodwall. The overtopping rate is therefore considered to be a better physical quantity for hydraulic design than the wave runup. A more fundamental argument is the deterministic approach of the FEMA guideline. This issue is dealt with in the next section.

Hughes (2007) carried out a literature survey to underpin the value for the overtopping criterion for levees that must be applied during design. The survey shows that various numbers have been proposed although the experimental validation of these numbers is very limited. Typical values according to the Dutch guidelines are (see also TAW, 2002):

- 0.001 cfs/linear ft (cfs/ft) for sandy soil with a poor grass cover;
- 0.01 cfs/ft for clayey soil with a reasonably good grass cover;
- 0.1 cfs/ft for a clay covering and a grass cover according to the requirements for the outer slope or for an armored inner slope.

The literature review suggests that a 0.1 cfs/ft is an appropriate range for maximum allowable overtopping rates based on Dutch and Japanese research for a well-maintained grass-covered levee.

However, it is difficult to assess the adequacy of applying the proposed TAW-criteria for the New Orleans area without a good understanding of the overall quality of the levees following many different periods of construction and the effects of stresses of past hurricanes. The actual field evidence supporting these criteria is limited. Based on consultation from a wide variety of experts, the following overtopping criteria have been applied for levees and floodwalls in this design report:

- Limit overtopping to average 0.1 cfs/ft at upper 90% confidence limit and to average 0.01 cfs/ft at 50% confidence limit for grass-covered levees
- Limit overtopping to average 0.1 cfs/ft at upper 90% confidence limit and to average 0.03 cfs/ft at 50% confidence limit for floodwalls with appropriate protection at the backside

The 90% confidence limit is chosen to be in line with the current Corps guidance on levee certification, see next section for further details.

The overtopping criteria are considered to be based on the best available current knowledge (2007) and are accepted to be applied to the Hurricane Protection System around New Orleans after consulting an ASCE review team. Comprehensive field tests will be carried out in the near future to determine the most appropriate values. These design criteria may change based on future research (e.g. field experiments) and are not necessarily applicable to other coastal areas.

3. PROBABILISTIC DESIGN METHOD

A more fundamental problem with the current FEMA certification guideline for coastal protection is the deterministic approach. The current guideline includes an extra 1ft of freeboard to the design height to account for uncertainties. A disadvantage of this concept is that the uncertainty in the various variables is likely not be constant but is a function of the uncertainties in water level and wave characteristics at a specific location. Using the current guideline may result in a design that is too conservative for one place and too optimistic for another location depending on the spatial variability in uncertainties.

The Corps guidance on certification is for a probabilistic approach to be followed similar to the approach used in Corps planning studies (USACE, 2006). Figure 2 shows the levee certification decision tree based on this guidance. Assurance plays a central role in this decision tree. It is defined herein as the percent chance non-exceedance given the 1% chance annual event occurs. The existing guidance, however, is not very specific as how to integrate all uncertainties associated with the surge and waves, and in case of levee certification, clarification as to how determine if levee protection is 90% assured.

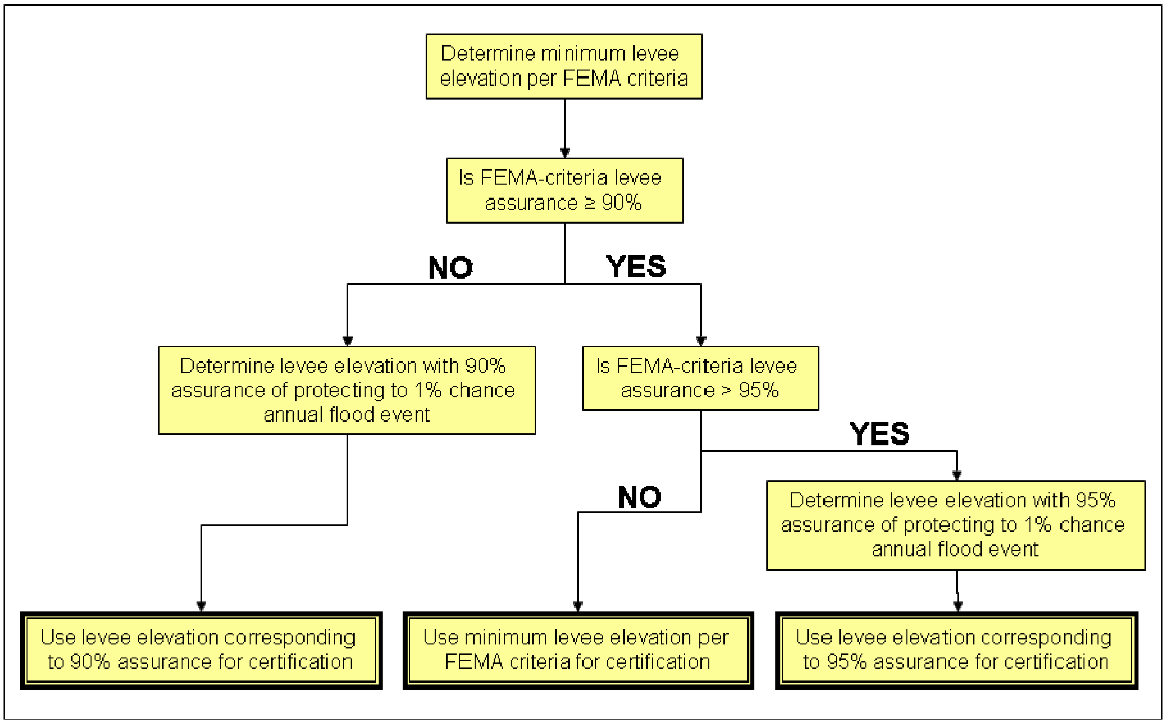

Figure 2. Corps' Levee Certification Decision Tree based on risk-based analysis (USACE, 2006).

As a first step, a probabilistic method is proposed herein to include these uncertainties in an explicit way. The probabilistic design method adopted herein follows the well-known Monte Carlo Simulation (MCS) technique. Basically, a MCS consists of three consecutive steps:

- Define the **input** variables in terms of probability density functions (and their possible correlations)
- Evaluate a large number of input combinations in an automated **process**
- Assess the **output** in terms of probabilities

In our case, the design output variable is the overtopping rate. To determine the overtopping rate, the probabilistic overtopping formulations from Van der Meer are applied herein. For convenience, these formulations are summarized in Textbox 1 (see e.g. TAW, 2002). Note that empirical formulations are not necessary for this uncertainty analysis. For instance, Boussinesq results could be also incorporated (see USACE, 2007).

The **input** variables for the overtopping formulation of Van der Meer are: geometrical parameters (levee height and slope), hydraulic input parameters (water elevation, wave height, wave period), empirical coefficients, and influence factors (berm factor, wall factor, wave angle factor, etc). We neglect the uncertainties in the geometrical parameters and influence parameters for reasons of simplicity, and only focus on the hydraulic input parameters, and the empirical coefficients. The mean and standard deviation of the empirical coefficients are known (see Textbox 1). These coefficients are assumed to be normally distributed and uncorrelated.

Textbox 1: Overtopping formulations

The overtopping formulation from Van der Meer reads (see TAW, 2002):

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b \xi_0 \exp\left(-4.75\frac{R_c}{H_{m0}}\frac{1}{\xi_0 \gamma_b \gamma_f \gamma_\beta \gamma_v}\right)$$

$$with \quad \max imum: \quad \frac{q}{\sqrt{gH_{m0}^{3}}} = 0.2\exp\left(-2.6\frac{R_c}{H_{m0}}\frac{1}{\gamma_f \gamma_\beta}\right) \qquad (1)$$

With:

q : overtopping rate [cfs/ft]

g : gravitational acceleration [ft/s^2]

H_{m0} : wave height at toe of the structure [ft]

ξ_0: surf similarity parameter [-]

α : slope [-]

R_c : freeboard [ft]

γ : coefficient for presence of berm (b), friction (f), wave incidence (β), vertical wall (v)

The coefficients -4.75 and -2.6 in Eq. 1 are the mean values. The standard deviations of these coefficients are equal to 0.5 and 0.35, respectively and these errors are normally distributed (see TAW document).

Eq. 1 is valid for $\xi_0 < 5$ and slopes steeper than 1:8. For values of $\xi_0 > 7$ the following equation is proposed for the overtopping rate:

$$\frac{q}{\sqrt{gH_{m0}^{3}}} = 10^{-0.92}\exp\left(-\frac{R_c}{\gamma_f \gamma_\beta H_{m0}\left(0.33+0.022\xi_0\right)}\right) \qquad (2)$$

The overtopping rates for the range $5 < \xi_0 < 7$ are obtained by linear interpolation of eq. 1 and 2 using the logarithmic value of the overtopping rates. For slopes between 1:8 and 1:15, the solution should be found by iteration. If the slope is less than 1:15, it should be considered as a berm or a foreshore depending on the length of the section compared to the deep water wave length. The coefficients -0.92 is the mean value. The standard deviation of this coefficient is equal to 0.24 and the error is normally distributed (see TAW, 2002).

Regarding the hydraulic input data, we use the best estimate 1% values for the water level, the significant wave height and the peak period based on the results of the Joint Probability Method with Optimal Sampling (see Resio et al., 2007). This method also provides the standard deviation in the 1% surge elevation. Standard deviation values of 10% of the average significant wave height and 20% of the peak period are used based on expert judgment of the current accuracy of the wave model (Smith, pers. comm.). Due to absence of data, all uncertainties are assumed to normally distributed. Furthermore, the errors in the surge and the wave characteristics are assumed to be uncorrelated, whereas the errors in the wave height and wave period are fully correlated. The reason behind is that the surge levels and the wave characteristics are obtained with different numerical models.

The **procedure** to evaluate the overtopping in a probabilistic way is as follows:

1. Draw a random number between 0 and 1 to set the exceedence probability p.
2. Compute the water elevation from a normal distribution using the mean 1% surge elevation and standard deviation as parameters and with an exceedence probability p.
3. Draw a random number between 0 and 1 to set the exceedence probability p.
4. Compute the wave height and wave period from a normal distribution using the mean 1% wave height/wave period and the associated standard deviation and with an exceedence probability p.
5. Repeat step 3 and 4 for the three overtopping coefficients independently.
6. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients determined in step 2, 4 and 5
7. Repeat the step 1 – 5 a large number of times (N)

The procedure is implemented in the numerical software package MATLAB.

The results from the procedure described above are analyzed to obtain the final **output** of the probabilistic analysis. The evaluation results in N overtopping rates. These overtopping rates are ranked from low to high and a probability is assigned to each overtopping rate using p = i/(N+1) with i = 1…N. The overtopping rate at a 50% confidence level is established by selecting the overtopping rate at p = 0.5. Similarly, the overtopping rate with 90% confidence level can be obtained at p = 0.9.

4. TEST CASES

This section presents several examples of the application of the probabilistic design method. For this purpose, a (hypothetical) base case of a levee section is defined to show various characteristics of the method, and a comparison will be made with the deterministic method. The hydraulic boundary conditions for this base case are chosen as follows:

- mean / standard deviation water level μ_ζ = 10ft / σ_ζ = 1ft
- mean / standard deviation significant wave height μ_{Hs} = 4ft / σ_{Hs} = 0.4ft (10% of mean)
- mean / standard deviation peak period μ_{Tp} = 8s / σ_{Tp} = 1.6s (20% of mean)

Furthermore, this levee section has a 1:4 slope and a wave berm at the still water level (berm factor γ_b = 0.6). All other influence coefficients (wave angle, vertical wall, etc.) are set to 1.0 which means no effect. These conditions are typical for the 1% conditions along the Lake Pontchartrain Lakefront area (see Figure 1). Figure 3 depicts the probability density function of the overtopping rate for the base case with the probabilistic method. The design height needed to meet the design criteria is 17ft in this specific case.

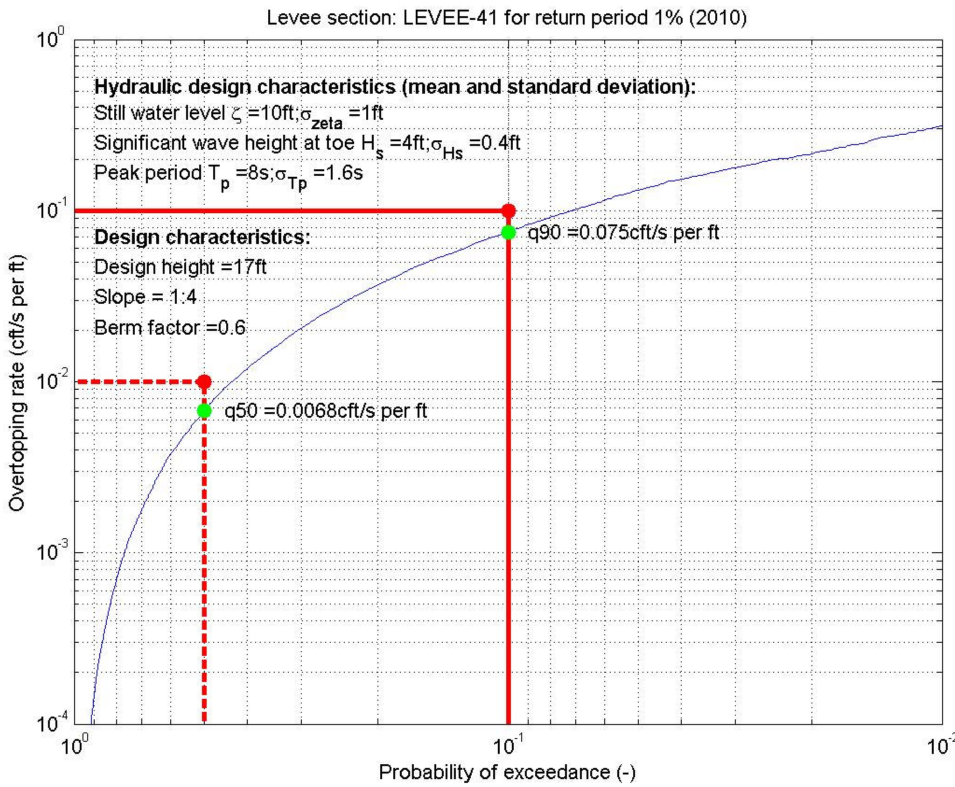

Figure 3. Output result of the probabilistic method for the base case

First, the number of simulations required to obtain statistically stationary results is investigated. To that end, the base case has been evaluated for three different settings of the error in the water level (0.5ft, 1ft and 2ft). The error in the 90% - overtopping rate has been determined for the raw output data of the MCS, see Figure 4. The error is defined as the relative difference between the 90% - overtopping rate based on N results and N-1 results. It shows that N = 10,000 results in an error of less than 0.5% for all three settings. Obviously, a lower initial standard deviation in the water level requires a smaller number of simulations to get similar accuracy.

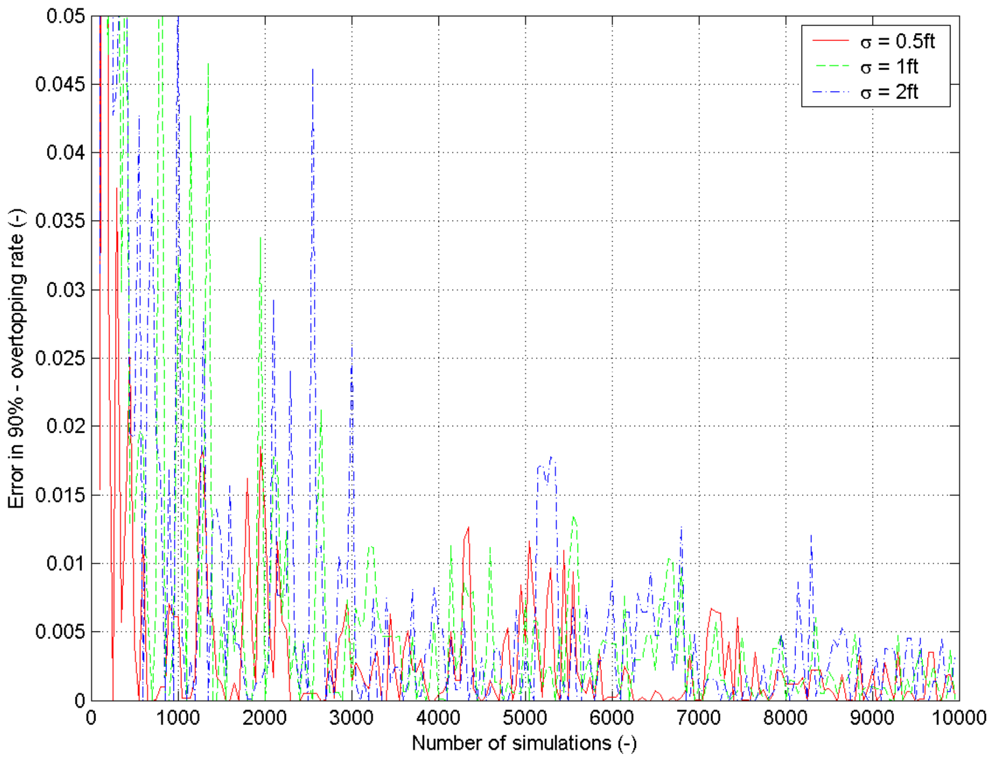

Figure 4. Sensitivity of accuracy in 90% - overtopping rate as a function of number of simulations for different errors in water level

The assumptions regarding correlation between the various hydraulic variables are also worth investigating. As stated before, the error in the water level and the errors in the wave characteristics have been assumed to be independent. The errors in the wave height and wave period, however, are fully correlated. These assumptions are weakly justified, and the sensitivity of these assumptions is analyzed. The base case is taken as a reference, and four other situations are also evaluated (see Table 1). The final design heights are rounded off at 0.5ft in upward direction following the common practice during the design process.

Situation	Overtopping rate (cfs per ft)		Design height (ft)	Correlation		
	50% value	90% value		Water level ζ	Wave height H_s	Peak period T_p
1. Base case (partial correlation)	0.007	0.078	17.0	No	Yes (T_p only)	Yes (H_s only)
2. Full correlation	0.004	0.090	18.0	Yes (H_s and T_p)	Yes (ζ and T_p)	Yes (ζ and H_s)
3. Partial correlation	0.008	0.090	17.0	Yes (H_s only)	Yes (ζ only)	No
4. Partial correlation	0.005	0.084	17.5	Yes (T_p only)	No	Yes (ζ only)
5. No correlation	0.007	0.063	17.0	No	No	No

Table 1. Effect of correlation between the hydraulic errors on the levee design height

Table 1 indicates that different assumptions with respect to the correlation between the various hydraulic errors can lead to differences of 1ft for this specific example. Logically, the situation with full correlation gives the highest design height, but this situation is highly unlikely. No correlation gives the lowest design height, but the difference with the base case is not significant. Note that also a full correlation between the water level and the wave height (situation 3) does not significantly differ from the base case, whereas a full correlation between the water level and the wave period (situation 4) raises the levee height with 0.5ft.

The third example focuses on the comparison between the FEMA certification guideline and the probabilistic approach presented herein (Table 2). The base case has been evaluated for different slopes (1/3, 1/4, 1/5, 1/6, 1/7) with and without a wave berm. The other variables are constant in this evaluation. The probabilistic method is evaluated with two different sets of overtopping criteria. Set 1 is 0.1 cfs per ft with 90% confidence limit and 0.01 cfs per ft with 50% confidence limit. Set 2 is 0.01 cfs per ft with 90% confidence limit and 0.001 cfs per ft with 50% confidence limit. The maximum wave runup that is included in the FEMA certification guideline is defined herein as the 2%-wave runup. This parameter is computed using the Van der Meer equation with the best estimate settings for the empirical coefficients (see TAW, 2002). The final design heights are rounded off at 0.5ft in upward direction and the results are summarized in Table 2.

Slope	Situation with wave berm ($\gamma_b = 0.6$)			Situation without wave berm ($\gamma_b = 1.0$)		
	FEMA guideline	Probabilistic method (Set 1)	Probabilistic method (Set 2)	FEMA guideline	Probabilistic method (Set 1)	Probabilistic method (Set 2)
1/3	22.0	18.5	22.5	23.5	20.0	24.0
1/4	19.5	17.0	20.0	23.0	19.5	23.5
1/5	17.5	16.0	18.0	22.0	19.0	22.5
1/6	16.5	15.0	17.0	20.5	18.0	21.0
1/7	16.0	14.5	16.5	19.0	17.0	19.5

Table 2. Comparison between design heights according to current FEMA certification guideline and probabilistic method with two different sets of design criteria (see text for further explanation)

The results in Table 2 show that the current FEMA certification guideline gives much higher design heights than the probabilistic method with the current design criteria (Set 1). The difference is about 1 – 3.5ft in this example. The design heights, however, are approximately equivalent for both approaches if the probabilistic results of Set 2 are considered. The FEMA certification guideline appears to be slightly lower than the probabilistic method in combination with Set 2 but the differences are small (0.5ft).

Finally, the effect of different mean values and errors of the still water levels and wave height is evaluated. The water levels are 10ft, 15ft, 20ft and 25ft with a standard deviation of 1ft, 2ft and 3ft. The mean wave height is equal to 40% of the water depth (i.e. 4ft, 6ft, 8ft, 10ft), and the standard deviation equals 10% of its mean value (i.e. 0.4ft, 0.6ft, 0.8ft and 1ft). The probabilistic method is evaluated with the current design criteria. The final design heights are rounded off at 0.5ft in upward direction and the results are summarized in Table 3.

Mean water level / mean wave height	FEMA certification guideline	Probabilistic method		
		Standard deviation water level 1ft	Standard deviation water level 2ft	Standard deviation water level 3ft
10ft / 4ft	19.5	17.0	18.0	19.0
15ft / 6ft	26.0	24.0	25.0	26.0
20ft / 8ft	32.0	30.5	31.5	32.5
25ft / 10ft	39.0	37.5	38.5	39.5

Table 3. Comparison between design heights according to the FEMA certification guideline and probabilistic method with different standard deviations of water level

Again, Table 3 indicates that the FEMA certification guideline results in higher design heights than the probabilistic method with the current design criteria. The difference between both methods logically decreases if the standard deviation of the water level increases. If the standard deviation in the still water level is 3ft, the resulting heights of both methods come close together. Note that this value for the standard deviation of the 1% water level is (highly) unlikely according to Resio et al. (2007). The influence of the wave height (and its associated standard deviation) on the difference between both methods is small. The difference is more or less constant for a fixed standard deviation.

5. DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

This paper presents a probabilistic method that explicitly takes into account the uncertainties in the various input parameters. The main advantage of this method is that this method does not include a (arbitrary) freeboard as it is the case in the current FEMA certification guideline. Another advantage is that the uncertainty in the water levels and the waves may differ from one area to another. This procedure enables the end-user to take this spatial variability into account in the hydraulic design process of flood protection systems.

The applications of the method show that the probabilistic method in combination with the current overtopping criteria (i.e. 0.01/0.1 cfs per ft at 50%/90% confidence limit) results in lower design heights than the current FEMA certification guideline. This difference is in the order of 1-3ft for the examples considered in this paper. If the overtopping rates in the current criteria are lowered with one order of magnitude (0.001/0.01 cfs per ft at 50%/90% confidence limit), the design heights of both methods are about equivalent. This is not surprising since there is a historical relationship between the 2%-runup and the overtopping rate of 0.001 cfs/ft (Van der Meer, pers. comm.). Note that our definition of the maximum runup from the FEMA guideline is the 2%-runup.

The application also highlights the strong dependency of the final levee heights on the choice of the allowable overtopping rates. The current overtopping criteria are largely based on expert judgment, and there exists no clear field experience to support these overtopping rates in coastal Louisiana. Comprehensive field tests are strongly recommended in the near future to determine the most appropriate values. Right now, the USACE is pursuing a testing plan for field experiments in New Orleans in the nearby future. Furthermore, a detailed analysis of overtopping rates and observed damage is recommended using the information during and after Katrina. This analysis would give valuable information about the allowable overtopping rates for different stages of levee damage.

Application of the probabilistic method requires extra information regarding the uncertainty in the various input parameters (error magnitude, distribution type) and correlation between the various parameters. Right now, the uncertainty of the wave characteristics is justified using expert judgment. Furthermore, the errors in the water level and the wave characteristics are assumed to be uncorrelated. One may argue that especially the error in the water level and the wave height are correlated because a somewhat higher water level might result in a higher wave height in shallow water. However, the effect of including this correlation seems to be small according to the sensitivity test.

Finally, it is worth mentioning that the probabilistic approach described in this paper essentially differs from a deterministic approach and a sensitivity analysis. A deterministic approach uses one (pre-selected) input combination and results in one output. In a sensitivity analysis pre-selected input combinations are evaluated to estimate the range of outputs. No formal insight is obtained in the probability of these output results in a sensitivity analysis. Using the MCS approach, hundreds (or thousands) of outputs are obtained and this gives insight how the input uncertainty is reflected in the output uncertainty. The development and implementation of this probabilistic method for the HPS around New Orleans is considered to be a necessary step forward toward a more rational way of including uncertainties into the design procedures of flood protection systems.

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